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Lada, Aleksandra; Ibsen, Lars Bo; Nicolai, Giulio

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**DCE Technical Memorandum No. 47**

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by

Aleksandra Lada  
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# The behaviour of the stiff monopile foundation subjected to the lateral loads

ALEKSANDRA LADA   LARS BO IBSEN   GIULIO NICOLAI

Aalborg University

Department of Civil Engineering

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## Abstract

The article focuses on the analysis of a large-diameter monopile foundation for offshore wind turbine based on the numerical model results. The case describes the behaviour of a monopile in sand subjected to a lateral loading conditions. The effects of the pile diameter, the length and the load eccentricity on the results are investigated. Additionally, the cyclic nature of load is also taken into consideration. The cyclic loading leads to an accumulated rotation, thus the SLS design must ensure that the maximum permanent rotation does not exceed the limit value for a given wind turbine. The approach analyzed by LeBlanc *et al.* [2010] and Lada *et al.* [2014] is discussed in the way of its use as a preliminary design.

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## 1 Introduction

The monopile foundation is the most often used concept for wind turbines in the offshore conditions. The greater demand for the renewable energy is the reason why bigger and more effective turbines are introduced in the offshore field. This has a significant influence on the foundation design. Not only the vertical load from the turbine itself are bigger, but also the overturning moment working cyclically on the pile increases. As the wind working on the rotor has its center at a greater height, the arm for the load increases.

Nowadays a large diameter monopiles are used for offshore wind turbines and they are considered highly successful. Nevertheless, the current design of monopile embedded in sand leaves a lot to be desired. The most often

used standards, DNV [2011] and API [2005] recommend the p-y curve design, which describes the non-linear relationship between the soil resistance and the lateral pile deflection at each depth below the seabed. The method is based on the full-scale tests of slender piles and it found its confirmation to a greater or lesser extent by other full-scale tests. However, the increase of the diameter was found by other researchers to change the behaviour of pile under lateral load and therefore the results of pile deflection do not fit the p-y curve any longer.

Currently, the attention of many engineers is devoted to the consideration of the cyclic loads in the design standards. Both, API [2005] and DNV [2011] recommend only the decrease of the ultimate soil resistance, when accounting for a cyclic loading. Nor the load characteristics, neither the number of cycles are included.

## THE STATE OF ART

The comparison between the p-y method and a fine element method for the response of a large-diameter pile subjected to a lateral load was performed by Lesny *et al.* [2007]. The models of two piles,  $D = 1$  m and  $D = 6$  m, embedded in non-cohesive soil were investigated. An elasto-plastic material behavior assuming the Coulomb friction law was the basis for the analysis. Both piles were designed by providing sufficient length for a rigid fixation and the limited pile head rotation,  $\alpha_{lim} = 0.7^\circ$ . As long as for smaller pile both results are quite comparable, for the large-diameter pile the results varies significantly. The p-y curve approach overestimates the pile-soil stiffness of a larger pile at a great depth, questioning the use of this method. Both piles were characterized by different diameters and embedded lengths, therefore it seems to be reasonable to state, that both these parameters influence the response of horizontally loaded piles.

In the research of Achmus *et al.* [2009] the numerical model was used in the analysis of laterally-loaded monopile. The results of drained cyclic triaxial tests on cohesionless soil was applied in the model, so the cyclic condition could be reconstructed. As the increase of plastic strain with the number of cycles was observed, it was interpreted as a decrease in soil secant stiffness. Therefore the model was named the degradation stiffness model. The soil was modeled as an elasto-plastic with a Mohr-Coloumb failure criteria and both, dense and medium dense sand were analyzed. Due to an investigation of different pile lengths, the difference in the behaviour of a stiff and a slender pile was observed. Due to applied cycles the increase of deformation and the lowering of the rotation point of a pile was noticed. From the results it was stated that the pile performance is very much dependent on the embedded pile length, whereas the increase of the pile diameter from 5 m to 7.5

m decreases the accumulated displacement of the pile only slightly. The design criterion considering the zero-toe-kick in order to minimize the risk of accumulated deformation under cyclic loading was claimed to be inadequate for the stiff piles. The piles with the same length, but different diameters were investigated and the stiffer pile that behaves more rigid had a smaller deformation on the mudline. Additionally, a strong dependence between the magnitude of the applied load and the accumulated displacement was found.

Many others publications describe the numerical models used in analyzing the response of laterally loaded piles. Some of them suggest the new expressions for p-y curves for the large diameter piles. Generally, a more rigid behaviour for less slender piles is proved. This results in the overestimation of the p-y curve stiffness on the great depth and cause also the underestimation of the rotation on seabed, [Abbas *et al.*, 2008],[Augustesen *et al.*, 2009],[Onofrei & Ibsen, 2010].

## SUBJECTS OF INTEREST

The numerical model has been made and the results of performed simulations are used to analyze the behaviour of a large diameter pile. The strong attention is also paid on the dependency of pile properties, like the diameter and the length and also the load eccentricity on the pile design.

In order to account for a cyclic loading a new approach based on LeBlanc *et al.* [2010] and Lada *et al.* [2014] accompanied with a numerical calculation is presented.

## 2 The numerical model of monopile

For numerical calculation a geotechnical program PLAXIS 3D is used, as it is able to simulate the soil condition with a good accuracy. The analysis is focused on the laterally loaded monopile embedded in a drained sand.

From a different available material models in PLAXIS 3D, the Soil Hardening Small Strain model is chosen, as a one of more advanced. The model assume the hardening of soil due to the plastic strains. The hardening due to compression and the hardening due to deviatoric loads (shear hardening) are distinguished in the model. Those two loads and also reloading of soil are controlled by different stiffness parameters,  $E_{50}$ ,  $E_{oed}$  and  $E_{ur}$ . The relation between the axial strain and the deviatoric stress is modelled to be hyperbolic. Additionally the model adopts the dependency between the stiffness and the stress level, by the parameter  $m$ .

In contrast to the basic Hardeing Soil model, a non-linear decrease of the stiffness due to the plastic strains is included. This change is controlled with the small-strain shear modulus,  $G_0$ , and the parameter providing the level of shear strain at which the reduction of the secant shear modulus to about 70 % is observed,  $\gamma_{0.7}$ . [Brinkgreve *et al.*, 2012]

### MATERIAL PARAMETERS

The advanced model requires a large number of input soil parameters in order to provide a realistic prediction of the soil behaviour. Three different state of sand are used in simulations: dense, medium dense and loose sand. The relative density,  $I_D$ , are chosen to be 85 %, 40 % and 5 % respectively. The required soil parameters are obtained from the data of the cone resistance,  $q_c$ . There is no specific location chosen and no data from CPT available, therefore

the data of  $q_c$  are evaluated based on  $I_D$ , equation (1) [Lunne *et al.*, 2007]. The atmospheric pressure,  $p_a$ , is set to be 100 kPa.

$$I_D = \frac{1}{2.96} \ln \left[ \frac{\frac{q_c}{p_a}}{24.94 \cdot \left( \frac{\sigma'_{v0} \cdot \left( \frac{1+2 \cdot K_0}{3} \right)}{p_a} \right)^{0.46}} \right] \quad (1)$$

The consolidation of the sand is assessed in order to provide a reliable formulations for the rest of the parameters. The friction angle is obtained from relation in equation (2). The pore pressure is neglected, so the total cone resistance,  $q_t$ , is equal to  $q_c$ .

$$\varphi' = 17.6 + 11 \cdot \log \left( \frac{\frac{q_t}{p_a}}{\left( \frac{\sigma'_{v0}}{p_a} \right)^{0.5}} \right) \quad (2)$$

The stiffness parameters are given in the dependency on the oedometer stiffness,  $E_{oed}$ , obtained from equation (3) for normally consolidated sand and from equation (4) for over-consolidated sand. The secant stiffness  $E_{50}$  is related to the oedometer stiffness through the Poisson's ratio as it is indicated in equation (5) and the unloading-reloading stiffness,  $E_{ur}$ , can be approximated with equation (6).

$$E_{oed} = q_c \cdot 10^{1.09 - 0.0075 \cdot I_D} \quad (3)$$

$$E_{oed} = q_c \cdot 10^{1.78 - 0.0122 \cdot I_D} \quad (4)$$

where  $I_D$  is in %.

$$E_{50} = E_{oed} \cdot \frac{1 - \nu - 2\nu^2}{1 - \nu} \quad (5)$$

$$E_{ur} = 3 \cdot E_{50} \quad (6)$$

The shear modulus obtained from equation (7) is set as the small-strain shear modulus.  $\gamma_{0.7}$  is calculated according to recommendation for the program [Brinkgreve *et al.*, 2012].

$$G = 1634 \cdot q_c^{0.25} \cdot \sigma_{v0}^{0.375} \quad (7)$$

In order to avoid numerical instabilities, the value of cohesion is set to be 0.1 kPa. The list of soil parameters used in the programm are



presented in Table 2.1. The reference values for stiffness are chosen as the ones corresponding to  $\sigma'_{v0} = 100$  kPa.

**Table 2.1:** Soil parameters used in PLAXIS

$I_D$ [%]	5	40	85
$\varphi$ [°]	32.56	37.55	44.49
$e$ [-]	0.843	0.734	0.595
$\gamma$ [kN/m <sup>3</sup> ]	18.90	19.46	20.28
OCR [-]	0.67	1.43	3.98
$K_0$ [-]	0.462	0.485	0.788
$E_{50}^{ref}$ [MPa]	18.30	103.520	142.60
$E_{oed}^{ref}$ [MPa]	25.84	132.61	150.00
$E_{ur}^{ref}$ [MPa]	54.91	310.57	427.80
$\nu_{ur}$ [-]	0.21	0.21	0.21
$G_0^{ref}$ [MPa]	69.04	93.25	177.00
$\gamma_{0.7}$ [-]	2.14e-7	1.71e-7	1.44e-7
$R_{inter}$	0.57	0.61	0.71

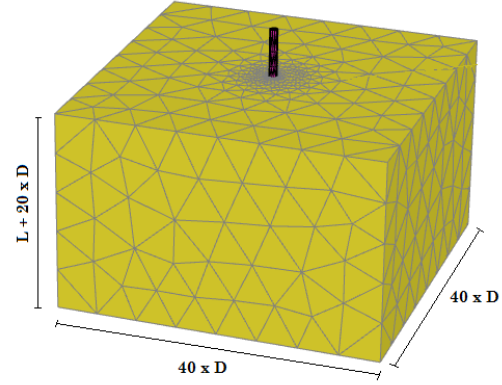
For the monopile a steel material is used. The stiffness is set to 210 GPa and the Poisson's ratio to 0.3. The wall thickness is chosen to be 0.06 m. The additional extension of pile is made in order to apply the horizontal load on the sufficient arm, creating the appropriate moment on the seabed. The stiffness of the pile extension is chosen to be much larger and its unit weight is chosen to be small, so there will be no effect of the additional part on the pile-soil interaction.

#### PLAXIS MODEL

The model boundaries was chosen based on the numerical model used by Abbas *et al.* [2008]. Chosen boundaries can be seen in Figure 2.1.

The model is divided into number of elements, so that the finite element calculations can be performed. Two different kinds of elements are used in the program to simulate an accurate soil behaviour and a pile behaviour. The third kind of elements are used for the

interface between pile and soil, what is really useful for investigating the soil-pile interaction. The parameters for soil on the interface are obtained by using the reduction factor  $R_{inter} = \tan(\delta)/\tan\varphi$ , where  $\delta$  is the angle of soil friction on the pile wall.



**Figure 2.1:** The model boundaries with the mesh example

The overall mesh is chosen to be coarse. As the most interesting results are expected near the pile, therefore the mesh is refined in the surroundings of the pile. There are two chosen region for mesh refining. The region closer to the pile is more refined. After performing a convergence analysis an appropriate Fineness-Factor is chosen for the soil volumes close to the pile and the pile surface. An example of meshed model is presented in Figure 2.1.

#### CALCULATION PHASES

The calculation for a laterally loaded monopile are divided into five calculation phases. The analysis is initiated by the initial phase, where the initial soil stress are obtained. In the phase a  $K_0$  procedure is used. This phase is followed by the nil-step phase, where no changes in activating parts is made, but the plastic calculations are used. The phase has no physical meaning and it is used only in order to ob-

tain an equilibrium of stress before applying the load. In the third phase, the construction phase, the plates are activated and in the next phase the vertical load is applied in order to simulate the self-weight of the wind turbine. In both phases the plastic calculation are chosen. In the final loading phase the horizontal load is additionally activated. The load is applied as a point load.

The determination of exact failure in the numerical calculation is found as difficult. Therefore the failure is assessed for a 4 ° of the pile rotation on the seabed. In case, where the calculation fails before obtaining the sufficiently large displacement, the tolerable numerical error is slightly increased.

### 3 The analysis of results

#### TEST PROGRAM

Different aspects of the pile behaviour are assessed and therefore different geometries and model properties are chosen for simulations. The test programs can be seen in Table 3.1

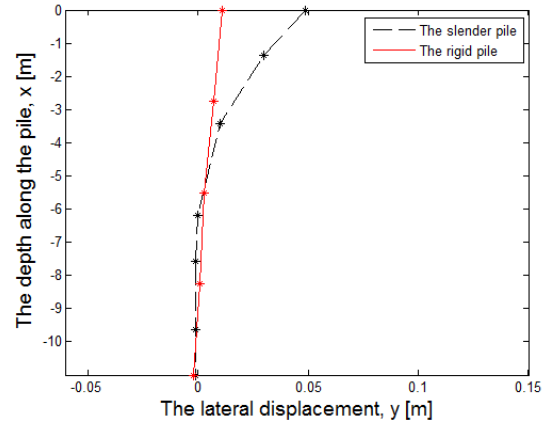
**Table 3.1:** Test program for PLAXIS simulations

Model	D [m]	L [m]	e [m]	$I_D$ [%]
1	1	11	1.2L	85
2	5	11	1.2L	85
4	6	25	1.2L	85
5	7	25	1.2L	85
6	5	30	1.2L	85
7	5	35	1.2L	85
8	5	25	1.0L	85
9	5	25	1.4L	85
10	5	25	1.2L	40
11	5	25	1.2L	5

#### PILE BEHAVIOUR

In order to assess the difference in the behaviour of the stiff and the slender pile the results of model 1 and 2 are investigated. In both cases the same length of the pile was chosen, but there is a difference in the pile diameter. The horizontal load is applied at the same height. The dense state of sand is chosen. The lengths of piles were chosen based on criterion proposed by Poulos and Hull (1989), where the pile behaviour is assessed as a rigid or flexible according to the soil and the pile stiffness and the pile geometry.

The ULS was found for both situations. The difference in the behaviour is analyzed when 30 % of the ultimate moment capacity is applied to the piles. The different pattern of deformation is presented in Figure 3.1.



**Figure 3.1:** The different pattern of behaviour for the slender (model 1) and the stiff pile (model 2)

Clearly, the slender pile gives the flexible response, whereas the rigid pile gives more stiff response. For the pile with the large-diameter there is a visible deformation on the pile top resulting in additional shear stresses. Even though the much larger force is applied for the stiff pile, the displacement on the seabed are significantly smaller. It can be concluded that the large diameter piles have a better lat-

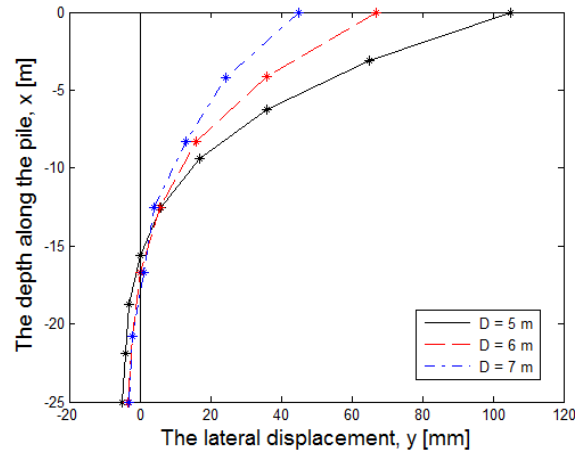
eral performance in comparison to the slender piles.

#### DEPENDENCY OF PILE PROPERTIES

The diameter of monopile is changed while keeping the others properties fixed. The effects of the diameter on the ultimate moment capacity on the seabed are presented in Table 3.2. The ultimate soil-pile resistance is important for the design, as it is used for the p-y curve. As the rotation of the pile on the seabed is often assessed for SLS, the change in the moment capacity on the seabed are interesting to inspect.

It can be clearly stated that the diameter has a significant effect on the moment capacity on the seabed. The diameter increase of 20 % gives a rise of more than 42 % in the moment capacity.

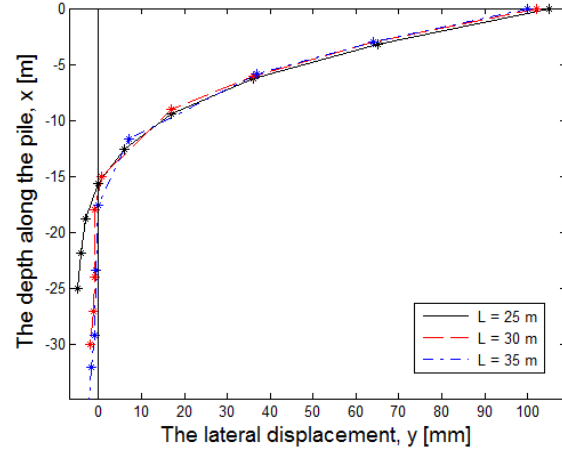
Additionally, the displacement of the pile, while applying 30 % of ULS from the main model (model no 3) are inspected in Figure 3.2.



**Figure 3.2:** Pile deformations for varying D,  $L = 25\text{ m}$ ,  $e = 1.2L$  -  $H = 19.3\text{ MN}$

It can be seen that the pile behaves more rigidly for all cases and the increase in the diameter gives a better pile performance.

The same analysis is performed for different pile lengths, while keeping other properties fixed. The results of the ultimate moment capacity can be seen in Table 3.3 and the deformation for the same applied horizontal force are plotted in Figure 3.3.



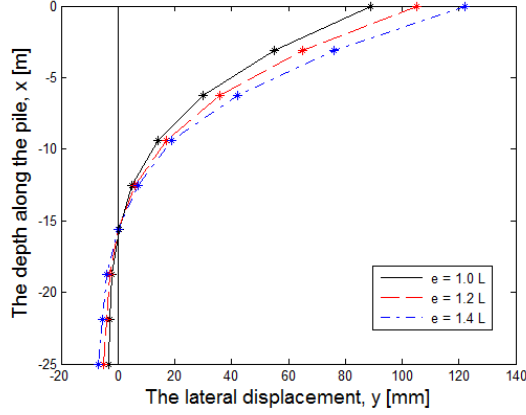
**Figure 3.3:** Pile deformations for varying L,  $D = 5\text{ m}$ ,  $e = 1.2L$  -  $H = 19.3\text{ MN}$

The increase in the length has an influence on the ultimate capacity by increasing it. The 20 % increase of length gives a 13 % increase of the moment capacity. It can be concluded that the change in the diameter is more effective for increasing the ultimate capacity. From the deformation graph it can be concluded that for applied 30 % of the ultimate moment, the difference in the displacement on the seabed does not vary significantly.

The load eccentricity is also inspected in order to assess its influence on the moment capacity. The results can be seen in Table 3.4. The change in the load eccentricity have also an effect on the ultimate capacity of the soil-pile system, however this parameter among others is assessed to be the least meaningful.

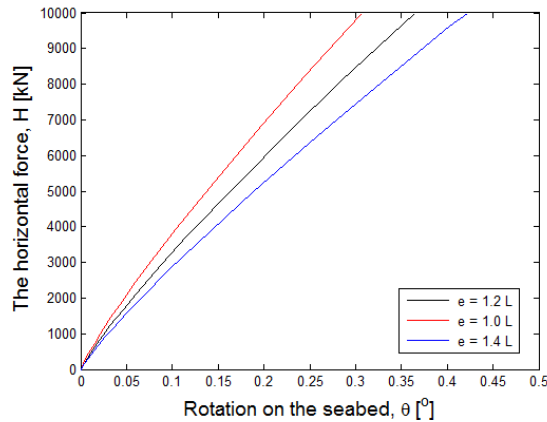
On the other hand, where the deformation of the pile for the same horizontal load is applied on the pile, it can be seen that the

load eccentricity has a bigger influence on the displacement on the seabed in comparison with the change of the length, Figure 3.4.



**Figure 3.4:** Pile deformations for varying  $e$ ,  $D = 5\text{m}$ ,  $L = 25\text{ m}$  -  $H = 19.3\text{ MN}$

As the change in the ultimate capacity is smaller, but the change in the deformation of the pile is more significant, the explanation for this is searched somewhere else. After inspecting the stiffness no change in the initial part is observed when the pile length is increased. When the load eccentricity is increased, the decrease in the rotational stiffness in its initial part is revealed. The results are presented in Figure 3.5.



**Figure 3.5:** The inspection of the initial rotational stiffness

**Table 3.2:** Results of the ultimate moment capacity for different pile diameters

Pile diameter, $D$ [m]	Moment capacity, $M_R$ [MNm]
5	1934.49
6	2754.50
7	3395.91

**Table 3.3:** Results of the ultimate moment capacity for different pile lengths

Pile length, $L$ [m]	Moment capacity, $M_R$ [MNm]
25	1934.49
30	2183.78
35	2262.05

**Table 3.4:** Results of the ultimate moment capacity for different load eccentricity

Load eccentricity, $e$ [m]	Moment capacity, $M_R$ [MNm]
1.0 L	1835.38
1.2 L	1934.49
1.4 L	2008.36

Finally the difference in the soil density on the ultimate capacity are inspected. The results for all three chosen relative densities are indicated in Table 3.5. The soil density is found to be the factor affecting the ultimate capacity significantly.

**Table 3.5:** Results of the ultimate moment capacity for different soil densities

Relative density, $I_D$ [%]	Moment capacity, $M_R$ [MNm]
5	467.24
40	1106.20
85	1934.49

## 4 The cyclic lateral load

LeBlanc *et al.* [2010] proposed a new method for determining an accumulated rotation for a long-term cyclic loaded monopile. Based on this research, the monopile response for lateral load in medium-dense and loose sand can be obtained. The approach was modified by Lada *et al.* [2014] for a dense sand. In both researches the cyclic test are described with the loading characteristics  $\zeta_b$  ( $\zeta_b = M_{max}/M_R$ ) and  $\zeta_c$  ( $\zeta_c = M_{min}/M_{max}$ ).

### ACCUMULATED ROTATION DUE TO THE CYCLIC LOADING

For the accumulated rotation an exponential dependency on the number of cycles was revealed. The displacement due to the cyclic loads can be approximated from equation (8).

$$\frac{\Delta\theta(N)}{\theta_s} = \frac{\theta_N - \theta_0}{\theta_s} = T_b \cdot T_c \cdot N^n \quad (8)$$

For medium-dense and loose state of sand the parameters  $n$  was chosen to be 0.31. For dense sand the value was found to be smaller and it differs between tests. However, the mean value can be chosen,  $n = 0.14$ . The dimensionless functions can be predicted based on Figure 4.1 and Figure 4.2.

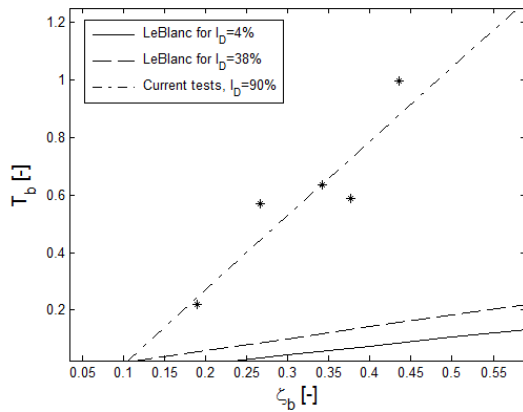


Figure 4.1: The dimensionless function  $T_b$

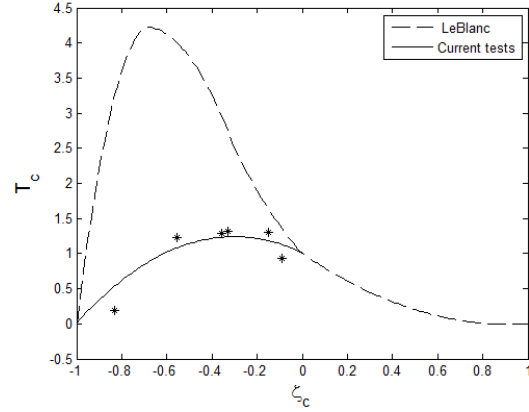


Figure 4.2: The dimensionless function  $T_c$

The approach can be used as a preliminary design, however it requires the information of the static corresponding rotation.

The ULS for three different state of sand is found in PLAXIS. The pile of  $D = 5$  m and  $L = 25$  m is chosen as an example. The horizontal load is applied on the arm of  $1.2 \cdot L$ . The FLS is assessed for 30 % of ULS, therefore corresponding load magnitude is applied and the rotation is obtained. The results are indicated in Table 4.1.

Table 4.1: The static rotation corresponding to 30 % of ULS

$I_D$ [%]	Static rotation [ $^\circ$ ]
5	0.446
40	0.603
85	0.837

### EXAMPLE

The load characteristics of  $\zeta_c = -0.9$  is chosen and the number of cycles is predicted to be  $10^7$ . The pile is embedded in dense sand. The accumulated rotation,  $\Delta\theta(N)$ , can be calculated as following:

$$\Delta\theta(N) = T_b \cdot T_c \cdot N^n \cdot \theta_s = 0.56 \cdot 0.4 \cdot (10^7)^{0.14} \cdot 0.837 = 1.79^\circ$$

The most often the tolerable criteria for the permanent rotation is set to be  $0.2^\circ$ , so the obtained value exceeds this limit. However, in this case all cycles are working in the same direction, and as the multi directional loading is rather expected, the permanent rotation would be smaller. Nevertheless, when  $\zeta_c$  is set to be  $\approx -0.4 \div -0.6$  the accumulated rotation would be much larger.

## 5 Conclusion

A number of simulations for the laterally loaded monopiles embedded in drained sand have been performed. Different pile geometries and model properties have been used in order to analyze the changes in the ultimate soil resistance. Currently, the most often used approach for the design of offshore monopile is the p-y curve approach. The relation between the pile lateral deformation and the soil resistance is based inter alia on the ultimate soil capacity. According to the standards, only the pile diameter and the soil properties have the influence on the value of the ultimate resistance. Moreover, the p-y curve was formed for a slender pile. Currently the large diameter piles that behave more rigidly are used in the offshore sector.

The results of PLAXIS simulations confirm that there is a difference in the deformation pattern between the slender and the stiff pile. The former behave flexible under the lateral loads, whereas the latter show a more rigid response. The results of different simulations reveal that the ultimate moment capacity is dependent not only on the pile diameter and soil properties, as it is stated in DNV [2011] and API [2005]. The length and the load eccentricity was found to influence the bearing resistance as well. However, the pile diameter and the change in the soil density are found to be the most significant, whereas the load ec-

centricity is considered the least contributing. Nevertheless, when the deformation of the pile for applied 30 % of ULS are inspected, the load eccentricity was found to be more meaningful than the length of the pile. This is explained with the changes in the initial stiffness when changing the load eccentricity.

A method for designing the offshore monopile should be improved, so that all the parameters influencing the pile displacements will be included.

Also the cyclic response for the monopile should be accompanied by the new calculation approach. The proposition given by LeBlanc *et al.* [2010] should be still investigated for its validation and confirm by the full-scale results.

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